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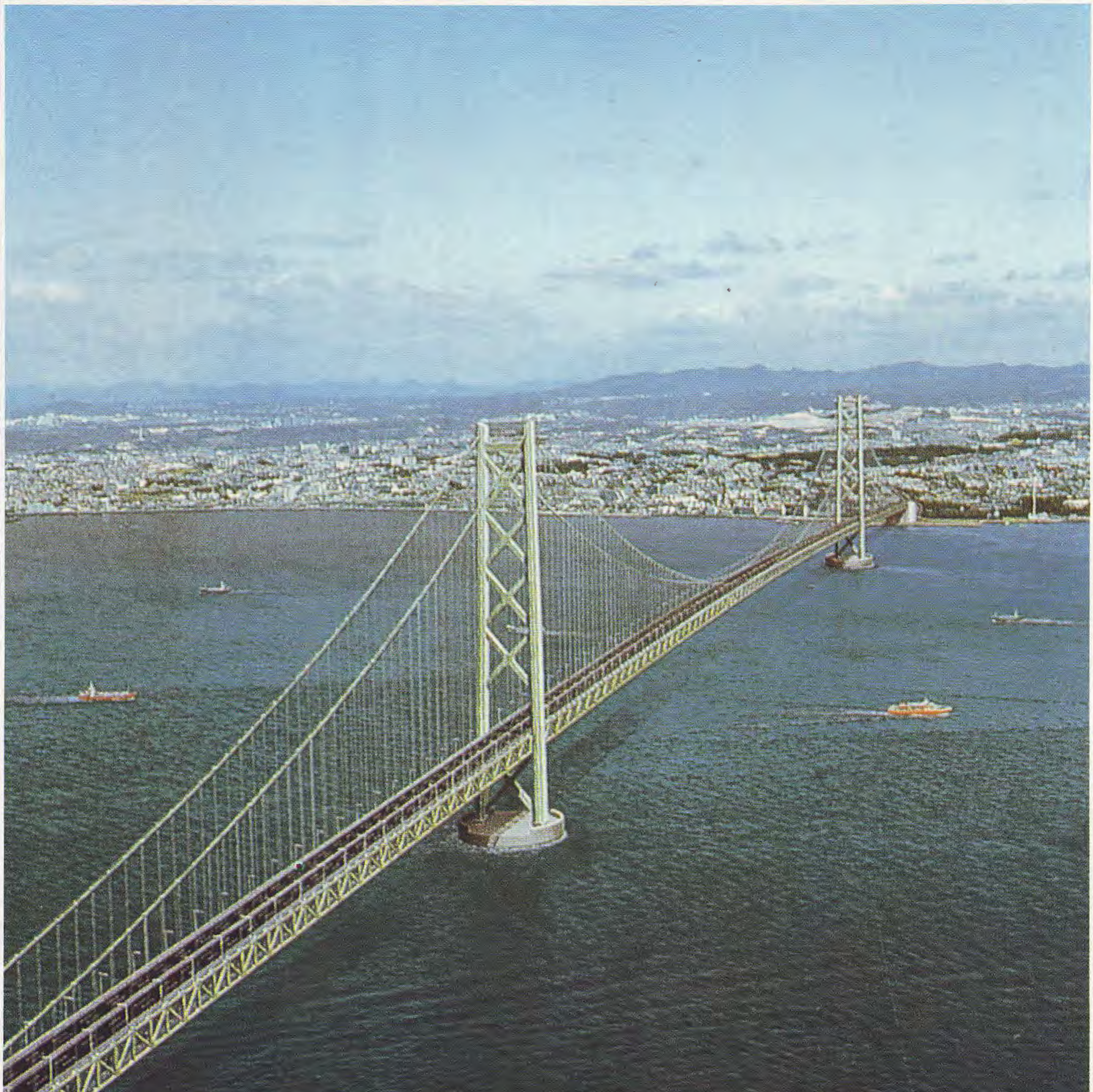

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Cover:
Photo shows the Akashi Kaikyo Bridge, Japan.

DESIGN BITUMEN CONTENT DETERMINATION FOR POROUS ASPHALT

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Abstract

Porous asphalt is generally classified under open-graded material. It was developed to eliminate splash and spray and to reduce the occurrence of aquaplaning. Porous asphalt is characterised by its high porosity and inter-connected pores. Unlike dense mixes, there is no established procedure to determine the design bitumen content of such open mixes.

This paper proposes a simple laboratory method to determine the design bitumen content (DBC) of a previous mix. An initial estimate of the required bitumen content was obtained using the FHWA procedure and the method based on aggregate surface area. However, a performance based design criteria is adopted to select the DBC and is based on the following measured properties :

- Resistance to Disintegration
- Resistance to Clogging
- The onset of Binder Drainage

A maximum abrasion loss of 30% as determined from the Cantabrian test, defines the lower limit of the DBC. The upper limit of the DBC is established based on the requirement to avoid binder drainage but maintaining sufficient permeability to withstand clogging. In the clogging test, the resistance to clogging of a specimen is expressed in a numerical term which is then correlated with the mix bitumen content corresponding to a selected poor drainage criterion. This defines the upper limit to the DBC which should not exceed the bitumen content corresponding to the onset of drainage.

Preamble

Porous asphalt was born in the United Kingdom in the 1950's in the wake of aquaplaning and skidding problems faced by fast moving aircraft traffic. The material was subsequently adopted for use of roads on similar traffic safety grounds. When used on highways, porous asphalt has proven to be an excellent material to reduce splash and spray, improve skid resistance, mitigate aquaplaning and an overall improvement in traffic safety.

Over the years had witnessed an extensive response and use of porous asphalt in the Continent. It is quite an irony that this material is only used on a trial basis in the UK at various experimental sites. Failure of a porous asphalt section on the M25 London Orbital merely three months after laying fuelled further doubts on its durability. The Malaysian situation presents a contrary scenario. Porous asphalt has been tried on a number of roads, highways and expressways. Even local authorities expressed their interest in the use of this material. It is interesting to note the patchy application of porous asphalt on a number of sites along the North-South Expressway. This form of porous asphalt application appears to be a unique Malaysian experience which should be documented.

The literature on the optimum bitumen content determination procedure for dense mixes is very extensive. Mix design methods in accordance to the Marshall, Hveem, Hubbard-Field or Leeds Mix Design Method are all well-known to highway engineers. The unsuitability of conventional mix design method for porous asphalt has been cited in many instances. Interested readers can peruse the work of Bucchi and Arcangeli (1990), Gemayel and Mamlouk (1998), Jimenez and Perez (1990) and Smith (1976) on this matter. Unfortunately, current literature with respect to adequate methods to determine the design bitumen content for porous asphalt is very scanty. However, a well-designed porous asphalt should principally exhibit high permeability and could better resist clogging and overcompaction.

Materials For Porous Mixes

Crushed gritstone was the aggregate type used on account of its hard, strong and durable character. A 14 mm nominal size was adopted to minimise the effect of container wall. The dried aggregates were blended by a sieving method and batched to gradations PR and BS whose particle size distributions are shown in Table 1. Gradation BS conforms to BS 4987 gradation limit for a 10 mm pervious wearing course (BSI 1988). Gradation PR is the equivalent new gradation developed by applying the theory of aggregate packing (Cabrera and Hamzah 1996).

Corrected %binder = %bitumen (2.65/SG_c) Equation (2)

where SG_c = The apparent specific gravity of the aggregate.

The experimental results of the FHWA method on the PR gradation is as follows:

Percentage oil retained = 2.01%

Percentage oil retained corrected
= 2.01 x(2.768)/2.65
= 2.1%

Surface constant of aggregate, Kc = 0.95

Specific gravity of aggregate = 2.77

Table 1: Aggregate Gradations Used in this Investigation

Gradation Designation	Cumulative Percentage Passing on Sieve (mm)						
	14	10	6.3	3.35	2.36	0.60	0.075
PR	100	82.4	56.0	-	20.1	15.9	4.5
BS	100	90-100	40-55	22-28	-	-	3-6

The type of bitumen used was a conventional 100 penetration grade of specific gravity 1.017 g/cc. All cylindrical specimens were compacted using the Marshall impact hammer.

The Design Bitumen Content

1 Exploratory Methods to Determine DBC

In the USA, the Federal Highway Administration (FHWA) method of mix design for open-graded mixes has been in place since January 1974. Its applicability to determine the DBC of porous mixes in this study was assessed on an exploratory basis. The detailed FHWA mix design procedure is available elsewhere (NCHRP 1978) In the FHWA method, the dried aggregates were immersed in an SAE 10 lubricating oil for 5 minutes and then drained for 15 minutes at 60°C. The amount of oil retained was then converted into the surface capacity constant of the coarse aggregate (K_c), after which the required bitumen content was calculated from Equation (1).

%binder = $2K_c + 4$ Equation (1)

For aggregates whose specific gravity differ significantly from 2.65, the modified Equation (2) is to be used instead.

Therefore, DBC = (2.0 x 0.95 + 4.0) x 2.65/ 2.77
= 5.6%

The principle of optimum bitumen content determination based on aggregate surface area is well establish for dense mixes. A number of surface area constants have been developed such as those by the Asphalt Institute (1990). Once the aggregate surface area is known, the required bitumen content is simply a function of binder film thickness after due regards to binder absorption by the aggregate. The surface area method firstly involved sieving the aggregate through a selected nest of sieves. The aggregate surface area was the sum of the product between the percentage passing each sieve and the corresponding surface area factor. For the PR gradation, the surface area determined using this method was found to be equal to 4.6m^2/kg. Based on the work of Hamdani (1983), an 11 micron film thickness was found adequate for porous mixes. This is somewhat thicker than binder film that coats dense mixes due to the need to offset rapid binder hardening in porous mixes.

Considering 1 kg of aggregate and 1.017g/cc bitumen specific gravity, then :

Quantity of bitumen = 4.6 x 104 x 11 x 10-4 x 1.017
= 51.46 g

Expressing the bitumen content as a percentage of the total mix, Therefore,

bitumen content = 4.9%

Assuming a 0.2% bitumen absorption,

then the DBC = 5.1%

2 Adopted Design Criteria

The adopted DBC was based on the following criteria :

1. A minimum bitumen content to resist mix disintegration resulting from surface forces generated by traffic, and
2. A maximum bitumen content to achieve high permeability and to avoid binder drainage.

The two criteria are in conflict with each other and the solution is based on a trade-off.

a) Lower Limit of the Design Bitumen Content

(i) The Cantabrian Test

The minimum bitumen content to resist mix disintegration was established using the Spanish Cantabrian test of abrasion loss. The objective of this test was to measure the resistance to disintegration. The test involved subjecting a Marshall specimen of known mass to abrasion in the Los Angeles drum, without steel spheres, to 300 drum rotations at 18°C. The mass of the specimen after the test was determined and the resistance to disintegration was expressed as a percentage of mass loss in relation to its initial mass.

The results are shown in Figure 1 for both gradations. As suggested by the pioneers of this test method (Jimenez and Perez 1990), the lower limit of the DBC corresponds to a maximum abrasion loss of 35% and equalled 3.8% and 3.9% for mix PR and BS respectively.

b) Upper Limit of the Design Bitumen Content

The upper limit of the DBC was determined either from the clogging test or the newly developed binder drainage test.

(i) The Clogging Test

The objective of the clogging test was to evaluate the clogging of a Marshall specimen loaded with a permeant every 24 hours until full clogging almost set in. The permeant consisted of a water-clay-slit solution, of known viscosity and having a 10% solids concentration. The solids composition of the permeant was 30% clay and 70% slit. Permeability was measured using a falling head water permeameter. To carry out the test, a 1200 ml of the clogging solution was filled into the perspex standpipe of the permeameter which had a rubber stopper secured in its bottom orifice. After the solution had settled, the rubber stopper was removed and the time taken for the permeant to fall between two designated points on the standpipe was noted. The specimen was then allowed to dry for 24 hours and the procedure repeated every 24 hours. The intrinsic permeability (K) during each loading cycle was determined using Equation (3).

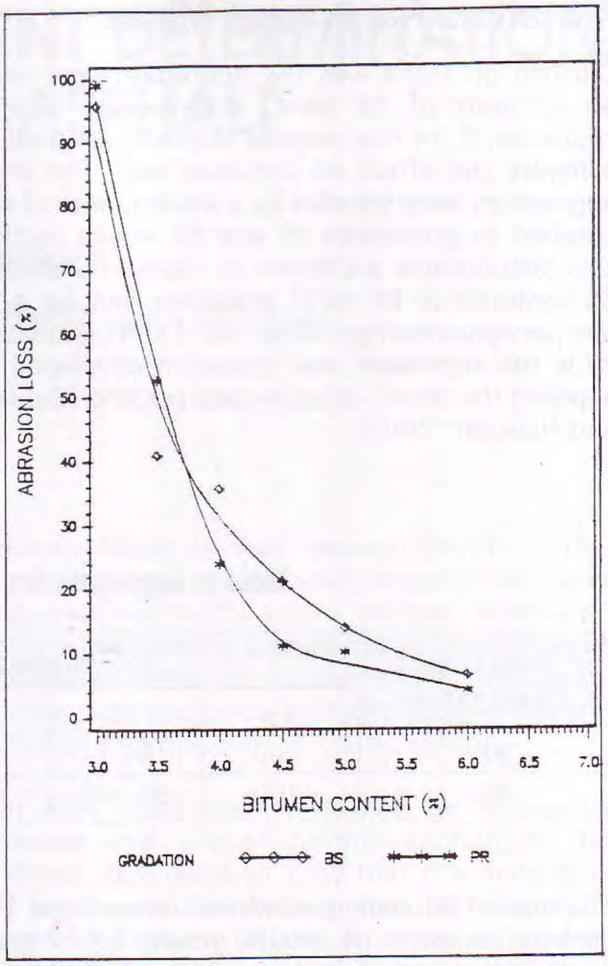

Figure 1: Results from the Cantabrian Test of Abrasion Loss on Mixes PR and BS

$$K = \frac{kn}{\rho g} \qquad \text{Equation (3)}$$

where K = Intrinsic permeability (m^2)

k = Coefficient of permeability (m/s)

n = Viscosity of permeant = 1.34 cP

p = Density of permeant = 1.053 g/cm^3

g = Acceleration due to gravity = 9.81 m/s^2

The reduction in K with every loading cycle is shown in Figures 2 and 3 for mix PR and BS respectively. The curves appear to converge to a point as clogging of the pores becomes severe.

The clogging test results are then expressed in terms of the relative change in K which is defined as the percentage difference between the initial K and the value of K at a particular loading cycle divided by the corresponding initial K. The relationship between the average relative change in K versus number of loading cycles are shown in Figures 4 and 5 for mix PR and BS respectively. Clearly, all curves originate from the origin. Nevertheless, for a given mix type, the curves lie close to each other and more importantly the DBC could not be inferred from this graph. Instead, the relative change in K of every individual specimen is calculated and plotted on the relative change in K versus loading cycle axes.

The next step is to express the degree of clogging in numerical terms and the procedure is graphically illustrated in Figure 6. Two tangents from the initial (T1) and upper (T2) portions of the curve were drawn. The intersection of T1 and T2 locates the co-ordinates I. It is desirable that T1 and T2 should be as shallow as possible so that point I shifts to as far right as possible. The horizontal distance (in days) between the origin to point I defines the overall number of cycles to voids closure due to clogging of pores. Cv and equalled 8.0 and 7.5 days for mix PR and BS respectively. The Cv of mix PR is higher than mix BS. This implies that mix PR sustains longer life than mix BS before clogging sets in.

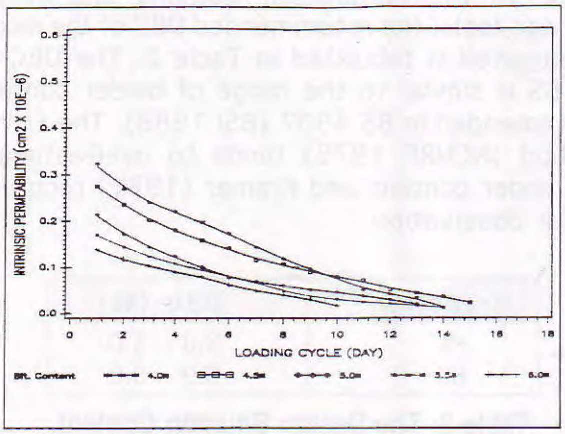

Figure 2: Reduction in Intrinsic Permeability versus Loading Cycles for Mix PR

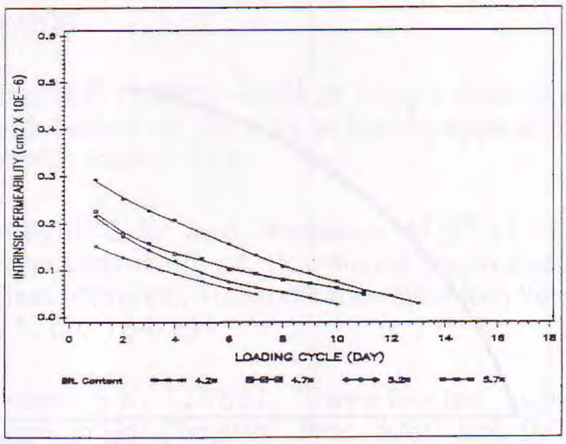

Figure 3: Reduction in Intrinsic Permeability versus Loading Cycles for Mix BS

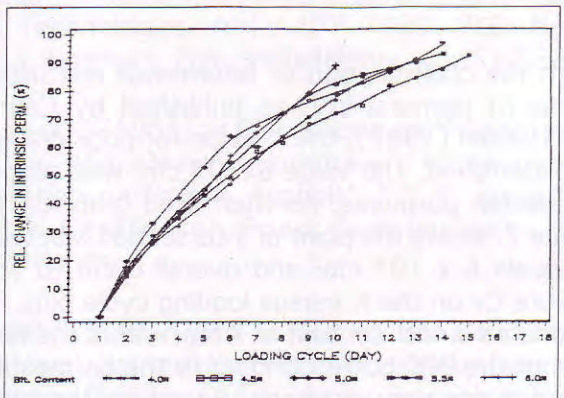

Figure 4: Relative Change in Intrinsic Permeability versus Loading Cycles for Mix PR

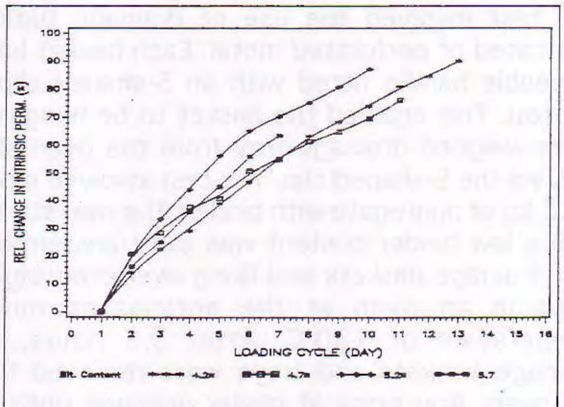

Figure 5: Change in Intrinsic Permeability versus Loading Cycles for Mix BS

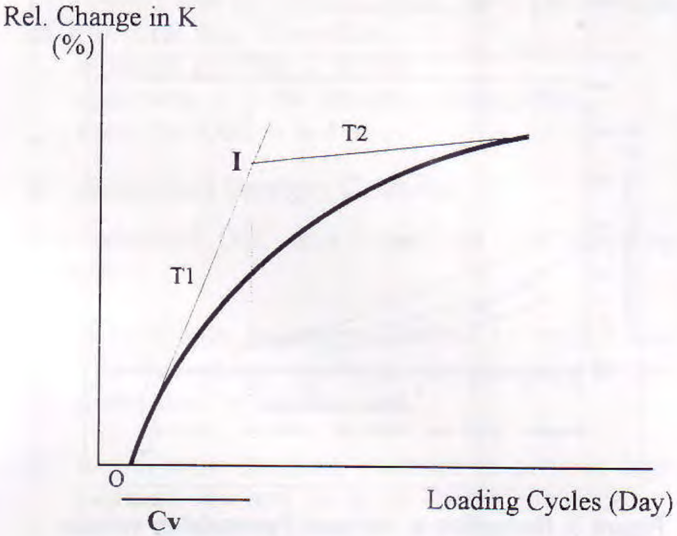

Figure 6: A Graphical Illustration to Express the Degree of Clogging in Numerical Term

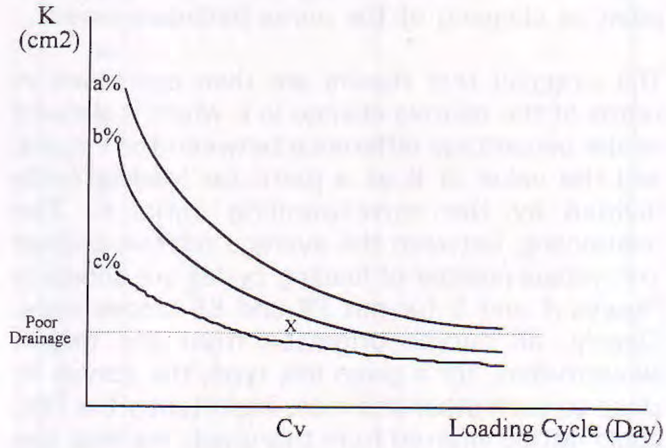

Figure 7: A Graphical Illustration to Determine the DBC Based on a Permeability Criterion

From the classification of bituminous mixtures in terms of permeability, as published by Cabrera and Hassan (1992), the K value for poor drainage was identified. The value 6×10^{-8} cm^2 was adopted for design purposes. As illustrated graphically in Figure 7, locate the point of intersection X between K equals 6×10^{-8} cm^2 and overall cycle to voids closure Cv on the K versus loading cycle plot. The estimated binder content at X represents the upper limit of the DBC corresponding to the permeability criterion for poor drainage. Based on the above procedure, the upper limit of the DBC for mix PR and BS are 5.0% and 5.2% respectively.

(ii) The Binder Drainage Test

The binder drainage test was carried out according to the TRL procedure described by Daines (1992). The test involved the use of drainage baskets fabricated of perforated metal. Each basket had a moveable handle fitted with an S-shaped clip at the top. This enabled the basket to be hung over a pre-weighed drainage tray from the oven steel rack via the S-shaped clip. The test involved mixing a 2.2 kg of aggregate with binder. The mix, starting with a low binder content was ten transferred to two drainage baskets and hung over pre-weighed trays in an oven at the anticipated mixing temperature of 130°C. After 3.5 hours, the drainage baskets and trays were removed from the oven. Any signs of binder drainage onto the tray was noted. The test was repeated in duplicate.

The method of determining the DBC based on this test differs from the TRL recommended procedure because the extent of drainage at binder contents up to 6.5% was low and it is unrealistic to adopt binder contents for porous mixtures in excess of 5.5%. Hence the criterion adopted was the onset or incidence of drainage.

For mix PR and BS respectively, the onset of drainage took place at 5.6% and 5.8% binder contents. The upper limit of the DBC from the clogging test should not exceed the binder drainage test results.

The Recommended DBC

Based on the Cantabrian, clogging and binder drainage tests, the recommended DBC of the mixes investigated is tabulated in Table 2. The DBC of mix BS is similar to the range of binder content recommended in BS 4987 (BSI 1988). The FHWA method (NCHRP 1978) tends to over-estimate the binder content and Kramer (1981) recorded similar observation.

Gradation	DBC (%)
PR	3.8 - 5.0
BS	3.9 - 5.2

Table 2: The Design Bitumen Content

Summary

A method of estimating the DBC of porous mixes has been proposed, eventhough an initial estimate is possible based on aggregate surface area computations. In this method, the binder content corresponding to 35% abrasion loss defines the lower limit. The upper limit of the DBC realistically considers a permeability rather than a porosity criterion. The adopted criterion is a better reflection of voids continuity. However, the binder content from the clogging test should not exceed the binder content established from the binder drainage test.

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MODIFIED RUBBERISED STONE MASTIC ASPHALT MIX FOR MALAYSIAN ROADS

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Abstract

Although material quality, mix design and construction practices are maintained to reasonably high standard in Malaysia, increasing traffic loading and severe environmental conditions warrant a new mix design concept to be developed. This paper looks into the possibility of using the stone to stone matrix type of mix (Stone Mastic Asphalt) on Malaysian roads. Several modifiers namely rubber, sulphur and butanol (SBR) were used to improve the performance of the mix. This study showed that the SMA mix is superior to the conventional mix in terms of stability, durability and strength.

Introduction

Stone Mastic Asphalt (SMA) is a stone to stone contact mix, whereby the coarse aggregates form a skeletal matrix that increases the stability of the mix. This interlocking system minimizes the lateral displacement of aggregates which is quite common to conventional hot mix asphalt mixes. The concept was first developed in Germany in the early sixties and was further developed in the United States in early 1990s (1).

One of the basic ingredients of SMA, besides its grading, is use a special type of cellulose fiber that forms a micro mesh netting to prevent drain-down of asphalt during storage, hauling and also laying (2). Asphalt drain-off or drain-down during prolonged hot pavement condition may contribute significantly in the propagation of surface distresses of flexible pavement. However this effect does not get notable attention in this part of the world including Malaysia.

In addition to the micro mesh netting, different types of additives were introduced to further enhance the mix. Among the additives used are polymers and rubber crumbs taken from shredded tyres. Other stabilising agents may also be introduced to further enhance this mix such as anti stripping agents such as lime. This paper presents preliminary results of mix performance of modified mix in SMA design.

SMA Mix Design

SMA is a high strength mix that requires good quality aggregates. Inferior quality aggregates may be crushed upon repeated loading which may inturn alter the stone matrix posture entirely. Therefore, the aggregates quality must be controlled to ensure the superior performance of mix.

Since Granite is abundantly available in Malaysia, it was identified to be the prime candidate for use SMA mix. In this research the Public Works Department specifications (3) for aggregate properties were adopted for formulation (Table 1). The gradation used (Figure 1) were specifically formulated to give higher stability and reliability with a maximum size of 16mm. Approximately 80% of the aggregates were larger than 2 mm, with 65 to 70 percent larger than 8mm .

The conventional 80/100 penetration asphalt is not suitable for use in SMA since it is soft. A lower penetration or stiffer asphalt was therefore used and this was achieved in several ways by blending suitable polymers, mineral rock wool or mineral fillers. Polymer - Modified Asphalt (PMA) would be ideal for use in Stone Mastic, but it is very expensive. Recent experiences in PMA (4) showed that it costs at least 300% more compared to the conventional mix. SMA with the right proportion of a technical raw material (Viatop-66 fiber) can be preblended with the 80-100 pen asphalt that would give a mastic behavior when used in SMA. In this research the 80/100 penetration asphalt was modified with ground tire

rubber, sulphur and Styrene Butadiene Random (SBR-Butonal) and binder characteristics of the asphalt used are summarized in Table 2.

for tire content of above 4%. The indirect tensile test also showed that mixes with 4% tire rubber, SMA4, had the highest resilient modulus. Since

Table 1: Aggregate Properties used for SMA Mix

No.	Type of Test	Results %	PWD Requirements
1	L.A Abrasion	19.70	≤ 30 %
2	Crushing Value	26.20	≤ 30 %
3	Impact Value	12.6	≤ 15 %
4	Soundness Test	1.76	≤ 15%
5	Polishing Stone Value	50.9	≥ 49
6	Flakiness and Elongation Test	17.1	≤ 20%
7	Specific Gravity	2.62	≥ 2.60
8	Water absorption	0.469	≤ 2%

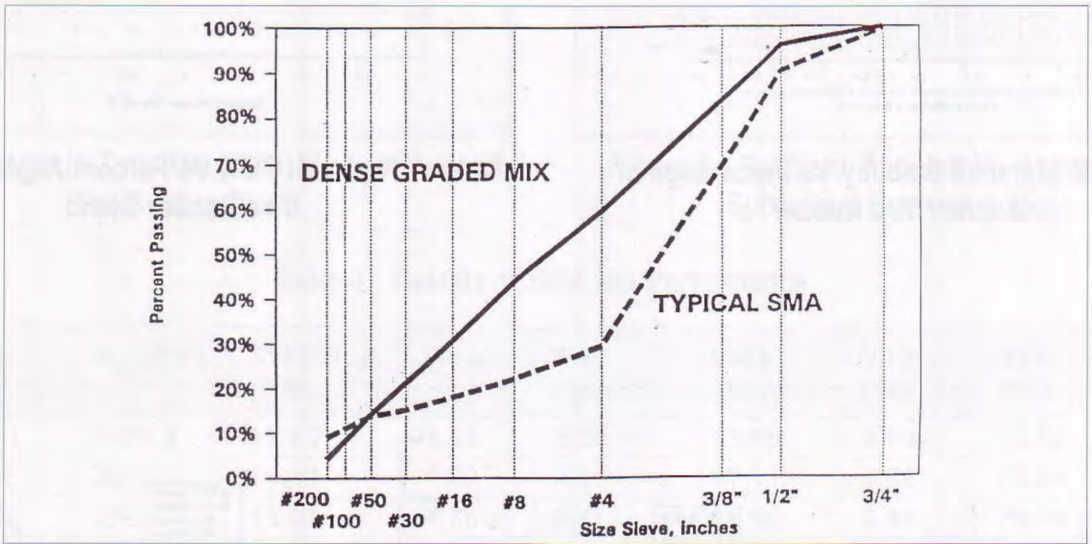

Figure 1: SMA and Conventional Gradations

Table 2: Rubberized Asphalt Physical Test Results

Asphalt Blend	Penetration (mm)	Softening Point °C	Thin Film Oven
A0 (80/100 pen-control)	85.7	52.3	0.003
A2 (A0+2% rubber)	79.3	63.3	0.009
A3 (A0+3% rubber)	71.5	68.2	0.010
A4 (A0+4% rubber)	62.0	70.3	0.011
A4-S (A4 + Sulfur)	61.7	70.3	0.009
A4-B (A4 + Butonal)	60.3	71.0	0.007

Optimum Asphalt Content

The performance of the rubberised abd control samples are shown in Figures 2 to 6 and Table 3. Samples with 4 percent tire rubber blended asphalt was found to have the desirable penetration for use in SMA. Higher percentage of tire rubber was tried but it was found to be very difficult to blend

SMA4 had the overall high mix performance value, two different materials (sulfur and Butanol) were used in the SMA4 blend to further enhance the properties . Both enhancers were added by weight of the preblended rubber-asphalt. The optimum amount of butonal and sulfur were determined based on the mix performance test results (Figure7).

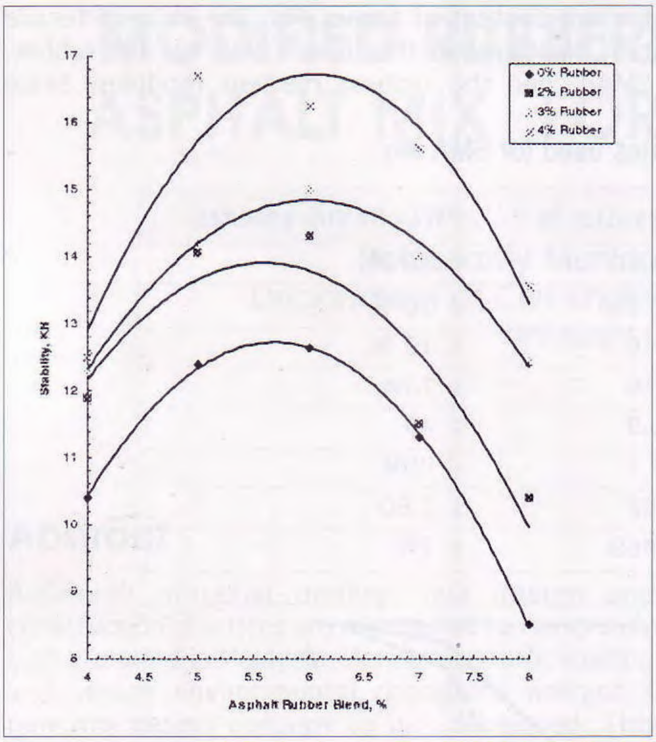

Figure 2: Marshall Stability Vs Percentage of Asphalt Tire Rubber

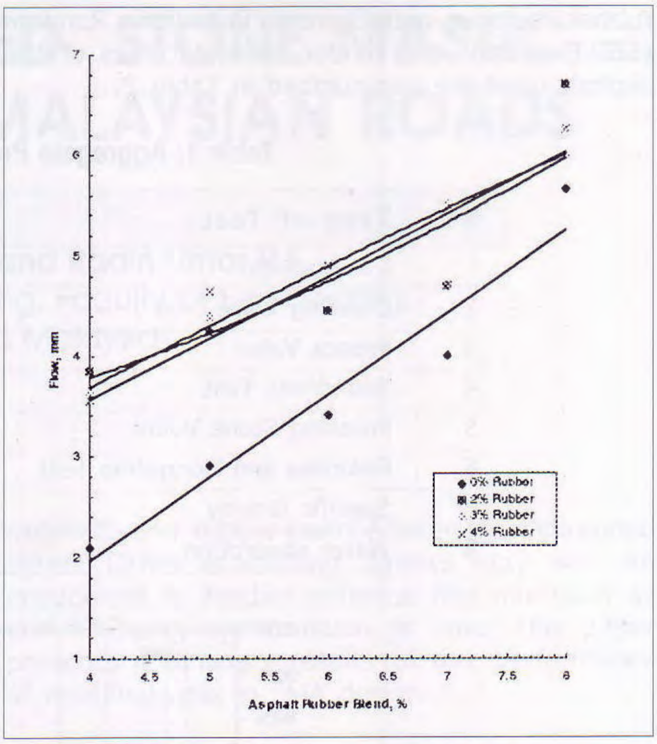

Figure 4: Flow Vs Percent Asphalt Tire Rubber Blend

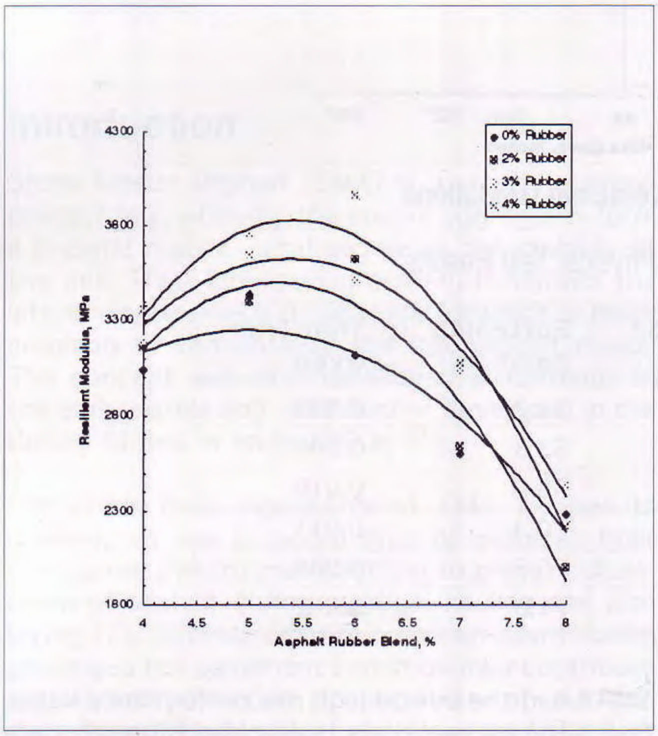

Figure 3: Resilient modulus Vs Percentage of Asphalt Tire Rubber Blend

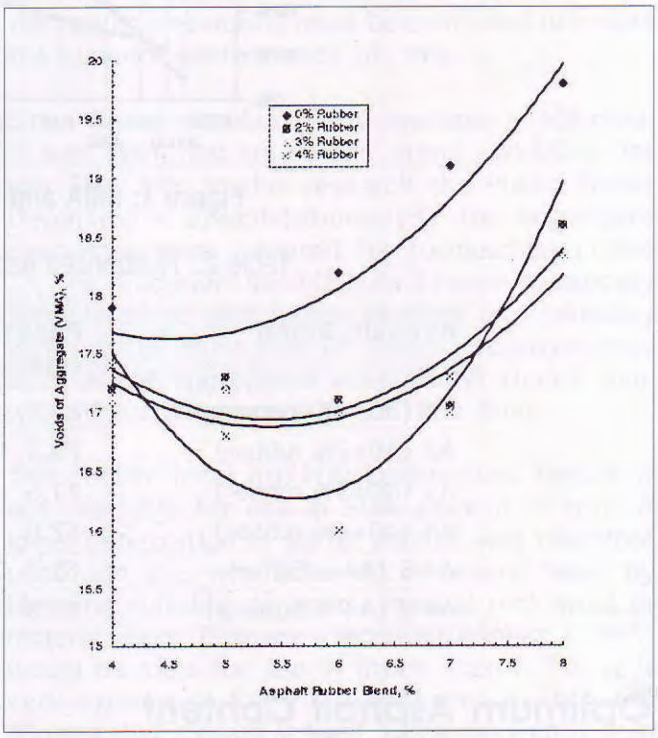

Figure 5: Voids in Mineral Aggregate(VMA) Vs % Asphalt Tire Rubber Blend

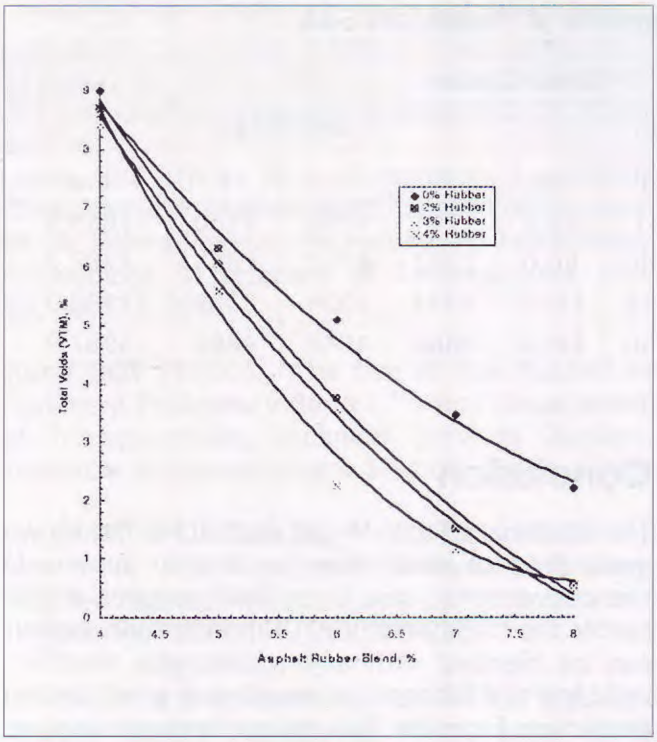

Figure 6: Voids in Total Mix (VTM) Vs % of Asphalt Tire Rubber Blend

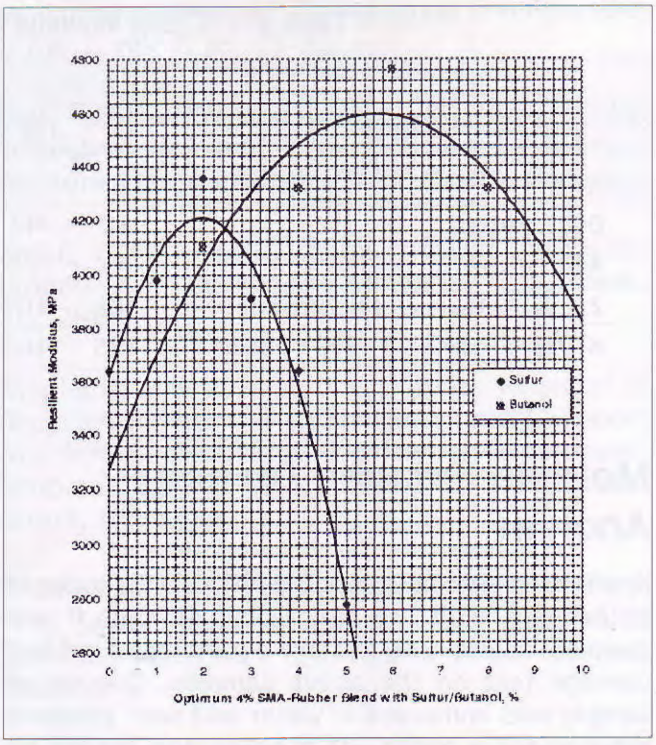

Figure 7 : Resilient Modulus Vs Percentage of Sulfur and Butanol at 25oC

Table 3 : Results of SMA Mix Performance

Mix	M_R(MPa)	Stability (kN)	Flow	Blk. Density	VMA (%)	VTM (%)	VFA (%)
SMA0	3436.3	12.6S	3.43	2.28	18.16	4.95	72.72
SMA2	3612.7	14.31	4.50	2.31	17.11	3.62	78.86
SMA3	3764.7	15.01	4.76	2.31	16.96	3.44	79.70
SMA4	3942.0	17.30	4.66	2.30	16.76	5.60	66.58
SMA4+ Sulphur	4275	17.88	4.60	2.35	17.82	4.95	72.31
SMA4+ SBR	4.885	18.31	4.69	2.35	17.79	5.04	69.87

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Dynamic Creep Performance

Dynamic creep modulus test was carried out on the samples using AusRoad Procedures to determine the number of load cycles to reach 1% , 3% strain and the ultimate failure (Table 4). It was found that for SMA4 it took 80 load cycles to reach 1% strain and 2782 load cycles to reach 3% strain while the control mix SMA0 displayed a much lower value of 68.3 cycles for 1% and 1855 cycles for 3% strain levels. With the addition of a small amount of sulfur and butonal separately showed a marked increase in the dynamic creep performance of the mix. The addition of sulfur (2.2% by weight of rubber-asphalt blend) increased the load cycles to 148 for 1% strain and to 3192 for 3% strain. However an addition of 5 percent butonal by weight of rubber-asphalt blend had the highest value. It took 5962 cycles to reach the 3% strain level and almost 20000 load cycles to failure. The control mix on the average failed at 5181 cycles. There was an increase of approximately 380% in creep performance.

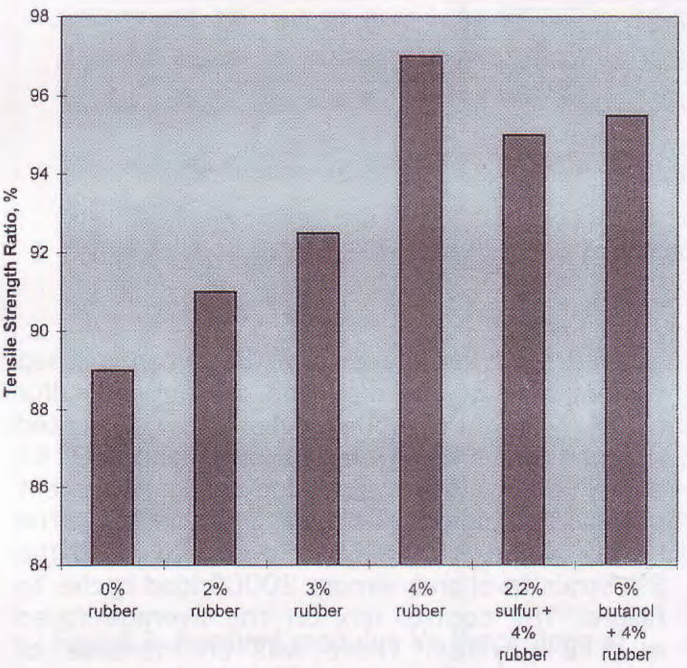
Table 4 : Creep Modulus Performance of Rubberized SMA

Sample	Load Cycles							
	1% Strain				3% Strain			
	1	2	3	ave	1	2	3	ave
0% rubber	52	92	61	68.3	1711	1755	2100	1855.3
4% rubber	79	71	90	80.0	2782	2712	2854	2782.7
2.2% sulfur + 4% rubber	180	105	158	147.7	3244	3004	3328	3192.0
6.0% butanol + 4% rubber	125	100	150	125.0	8360	4641	4885	5962.0

Moisture Induced Damage Analysis

Roads in tropical countries like Malaysia are exposed to extreme moisture conditions. As such it was deemed appropriate to carry out moisture induced damage test on the above samples. One set of sample was immersed in water and was pressure induced for a about 20 minutes and tested for Tensile Strength. Another set was tested dry without undergoing any moisture inducement. Figure 8 shows the Tensile Strength Ratio (TSR) of the SMA mix. It is simply the wet strength over the dry strength. The higher the TSR percentage, the higher the durability of the mix.

Figure 8: Tensile Strength Ratio of SMA Mix



Conclusion

The Rubberized Stone Mastic Asphalt Mix has shown great promise which may be able to supercede the conventional mix. Since SMA requires a stiff binder the traditional 80/100 penetration asphalt can be blended with any appropriate modifiers including tire rubber that would give good binding properties. From the SMA mix performance analysis it was found that 4 percent tire rubber gives the maximum strength and stability. A much higher stability and strength can be achieved by using special enhancers like Butanol and Sulfur. The traditional 80/100 binder can also be modified with cellulose fiber, which is an organic material. This results suggest that the SMA mix performance is superior to the conventional mix in terms of stability, durability and strength.

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DEVELOPMENT OF BACKCALCULATION ALGORITHM FOR LOAD TRANSFER EFFICIENCY OF JOINT

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Abstract

Load transfer efficiency of joint is one of the main factors influencing the structural capacity of concrete pavement. However, the evaluation of the load transfer efficiency of joint has not been well addressed. On the basis of an analytical model for a three-slab system, this paper developed a computer programme for backcalculating the load transfer efficiency of load using the method of least squares. It was founded that the program did not always produce the correct answers because it was sensitive to the choice of initial seed values of the unknown variables. Other computational issues related to the performance of the algorithm are also highlighted.

Introduction

For the purpose of backcalculation of load-transfer efficiency of joint, a multiple-slab model must be used. Most of the currently available models are based on the finite-element methods such as the KENSLABS programme (Huang 1993) and the FEACONC III computer programme (Tia et al. 1987). Although the finite element programmes are able to handle more realistic loading and boundary conditions, their application in backcalculation is limited. One of the major disadvantages to employ the finite methods models in backcalculation is that the accuracy of finite element methods controls the accuracy of backcalculation, especially the so-called iterative backcalculation methods. The accuracy of deflection calculation depends on the division of the finite element mesh. The choice of mesh sizes and layout is an important aspect of the solution.

Tia et al. developed a computer programme called DBCONPAS (1989) for a two-layer concrete pavement based on an iterative search. The DBCONPAS programme can be used to estimate the subgrade and concrete moduli and the edge stiffness utilising surface deflection measurements. However, the estimation of joint stiffness has not been developed at this stage.

In this paper, a three-slab model developed by Shi (1995) was evaluated and modified for use in the evaluation of the load-transfer efficiency of joint. The main advantage of the three-slab model over finite element models is that it provides analytical solutions rather than numerical solutions. This overcomes the problem arising from the use of finite element methods. in addition, the three-slab model can be easily modified and incorporated as a subroutine in the backcalculation programme. A backcalculation computer programme NUS-BACK4 was developed for evaluating the load-transfer efficiency of joint using the method of least-squares.

Description Of Three-Slab Analysis Model

1 Theoretical Assumption

The three-slab model Slab3-1 developed by Shi (1995) is shown in Fig.1. This model is based on the thick plate theory for a three-slab system supported on a Pasternak foundation. The slabs are represented by thick plates as defined by the Reissner's theory (1945). The Pasternak foundation model is shown in Fig. 2 and defined as follows (Pasternak 1954) :

$$p = kw - G_b \nabla^2 w \qquad \text{.................................(1)}$$

where p is the contact pressure on the surface of foundation, k is the modulus of subgrade reaction, w is the surface deflection of the foundation, G_b is the shear modulus of the foundation, and ∇^2 is

the Laplace operator. Note that Pasternak foundation is reduced to Winkler's foundation by setting $G_b = 0$.

Fig.1 Three-Slab System

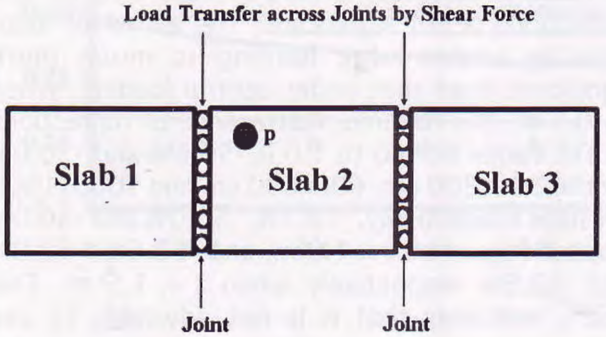
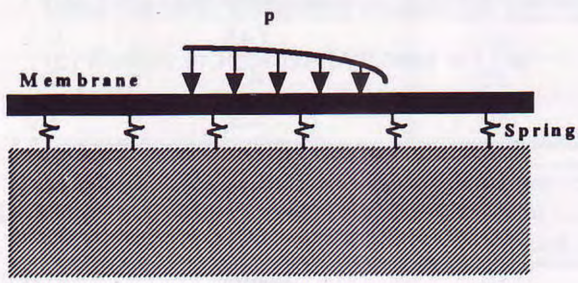


Fig. 2 Pasternak Foundation Model



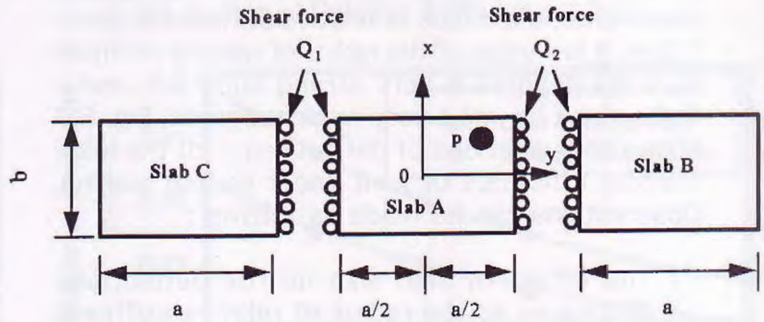
Only shear transfer is considered across the joints in the three-slab system. The shear transfer is assumed to be constant along a joint and is characterised by the load-transfer efficiency of joint JTE defined by

$$JTE = \frac{D_U}{D_L} \quad \text{...(2)}$$

Where D_U and D_L are deflections on both sides of the joint between the unloaded and loaded slabs. The value of JTE varies between 0 and 1. When JTE equals to zero, there is no shear transfer across the joint. When JTE equals to one, there is 100% shear transfer across the joint.

Based on the above assumption on shear transfer, the three-slab system shown in Fig. 1 can be discretized into three independent slabs A, B and C as shown in fig. 3. Slab A is loaded with an arbitrary load p and unknown shear forces Q_1 and Q_2 distributed along the two edges respectively; slab B is under the action of Q_2 and along one edge, and slab C is under the action of Q_1 along one edge.

Fig. 3 Three Discretized Slabs



Shi (1995(developed a programme named Slab-3 to compute pavement responses under loading using Microsoft FORTRAN-77. The Slab-3 programme is employed in this paper as a subroutine in the backcalculation of parameters of three-slab pavement systems. It should be pointed out that in the three-slab model, the properties of the three slabs, such as slab moduli, sizes and thicknesses are assumed to be identical. In addition, the load-transfer efficiencies of the two joints are also considered to be same. Therefore, errors would be observed in the evaluation of actual pavements in the cases where the above assumptions are not satisfied.

2 Effect of Load-Transfer Efficiency of Joint on Deflections

Shi (1995) has investigated the effect of load transfer across joints on pavement responses under loads using the three-slab model. He concluded that under central load, the effect of load transfer across joints should be considered when (a) the radius of relative stiffness is larger than 1.0 m; or (b) an edge or corner load is applied. Because of the significance of such effect in backcalculation, a parametric study was conducted using the slab size, radius of relative stiffness and load-transfer efficiency of joint as the variables. It should be pointed out that the radius of relative stiffness is a function of slab thickness, elastic modulus of slab and subgrade modulus. Two loading configurations were considered : (1) central loading, and (2) edge loading applied at the midspan of joint. The reasons for this consideration are to find out whether the deflections are sensitive to load-transfer efficiency of joints or not, and if the model can be used for deflection testing.

Nine cases of pavements were investigated as shown in Table 6.1. In the nine cases, three slab sizes were selected: 300x300 cm, 600x600 cm

and 1000x1000 cm. The values of radius of relative stiffness were 0.5 m, 1.0 m and 1.5 m. In actual pavements, the radius of relative stiffness is about 1.0 m. A low value of the radius of relative stiffness (0.5 m) denotes a very strong subgrade and a high value (1.5 m) a very weak subgrade. Fig. 6.7 shows the variations of deflections with the load-transfer efficiency of joint under central loading. Observations can be made as follows :

1) The effect of slab size on the deflections decreases as the radius of relative stiffness increases. For example, the difference of deflections between 300x300 cm and 600x600 cm is -5.3% when ℓ = 0.5 m, -18.7% when ℓ = 1.0 m and -43.4% when ℓ = 1.5 m.

2) The effect of load-transfer across joints depends on the radius of relative stiffness and the slab size. It is found that the load-transfer efficiency of joint has no significant effect on deflections when the radius of relative stiffness is low (0.5 m). Also, the effect of load-transfer on deflections varies with the slab size. When ℓ = 1.0 m, the difference of deflections between JTE=0 and JTE=1.0 is -9.30%, -1.5% and -0.1% for the 300x300 cm, 600x600 cm and 1000x1000 cm slabs respectively, and it is -23.9%, -4.4% and -1.0% respectively when ℓ = 1.5 m.

3) The radius of relative stiffness plays a big role in the three-slab system. As it increases, the deflection increases significantly, especially when the slab size is small. This is similar to the observation made in the analysis of a single slab (Westergaard 1926).

Fig.5 shows the variations of deflections with load-transfer efficiency of joints under edge loading. It is obvious that the trend of variations of deflections under edge loading follows the same trend as that under central loading. When the radius of relative stiffness is low, the effect of slab size on the deflections is not significant. The effect of load-transfer under edge loading is much more significant than that under central loading. When ℓ = 0.5 m, the maximum difference of deflections as JTE varies from 0 to 1.0 is -50.6% and -50.0% for the 300x300 cm, 600x600 cm and 1000x1000 cm slabs respectively, -52.3%, -50.0% and -50.0% respectively when ℓ = 1.0 m, and -52.6%, -52.0% and -50.5% respectively when ℓ = 1.5 m. This clearly indicates that it is not advisable to use central loading to evaluate the load-transfer efficiency of joints. Edge loading is the logical choice for this purpose.

Table 1

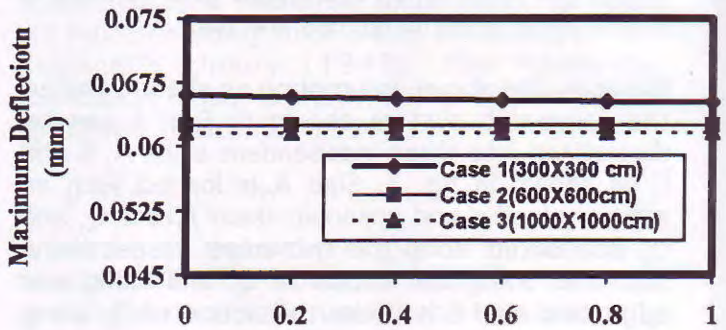
Case	Slab Size (cm)	Load (kN)	Thickness (mm)	Modulus		ℓ (m)
				Slab (GPa)	Sub (MN/m3)	
1	300x300	80	250	30	745	0.5
2	300x300	80	250	30	47	1.0
3	300x300	80	250	30	0.9	1.5
4	600x600	80	250	30	745	0.5
5	600x600	80	250	30	47	1.0
6	600x600	80	250	30	0.9	1.5
7	1000x1000	80	250	30	745	0.5
8	1000x1000	80	250	30	47	1.0
9	1000x1000	80	250	30	0.9	1.5

Note: l denotes the radius of relative stiffness and Sub denotes the subgrade

Fig. 4 Variations of Maximum Deflections with JTE under Central Loading

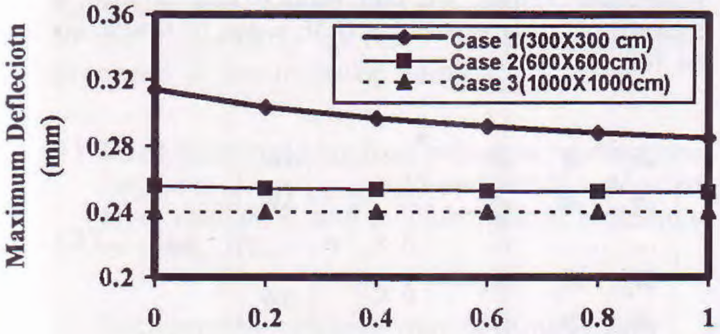
Load-Transfer Efficiency of Joint (JTE)

(a) Radius of Relative Stiffness = 0.50m



Load-Transfer Efficiency of Joint (JTE)

(b) Radius of Relative Stiffness = 1.0m



Load-Transfer Efficiency of Joint (JTE)

(c) Radius of Relative Stiffness = 1.5m

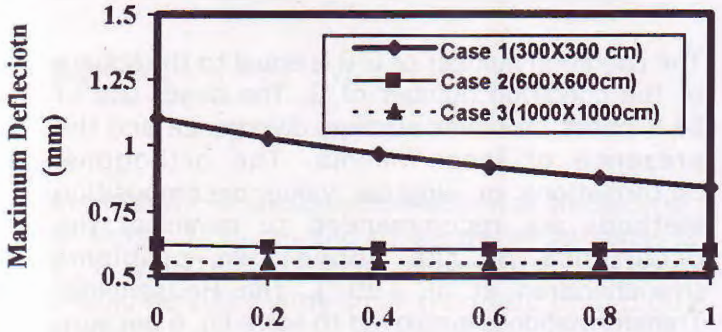
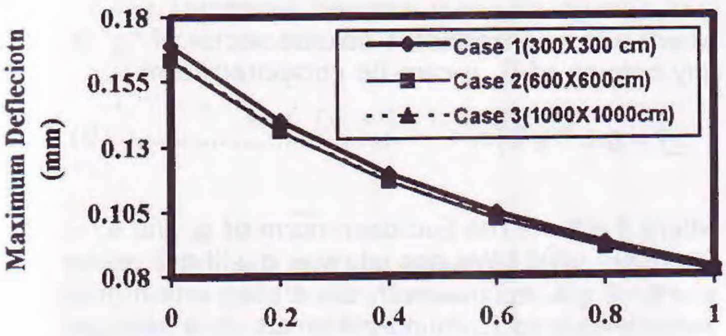


Fig. 5 Variations of Maximum Deflections with JTE under Edge Loading

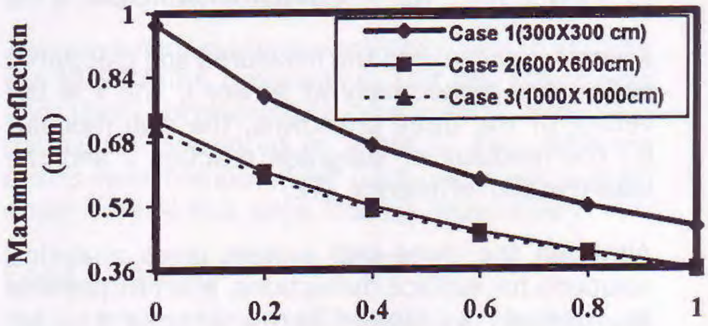
Load-Transfer Efficiency of Joint (JTE)

(a) Radius of Relative Stiffness = 0.5m



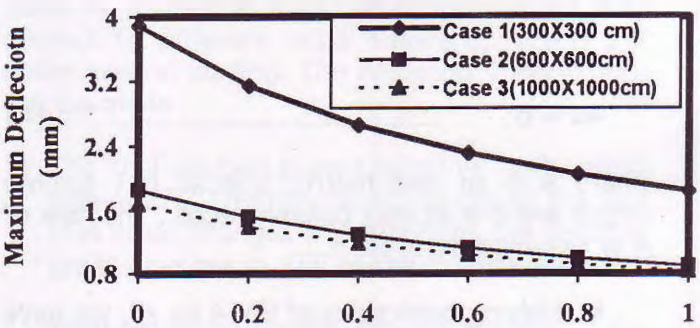
Load-Transfer Efficiency of Joint (JTE)

(b) Radius of Relative Stiffness = 1.0m



Load-Transfer Efficiency of Joint (JTE)

(c) Radius of Relative Stiffness = 1.5m



Development of Backcalculation Algorithm NUS-BACK4

1 The Least-Squares Problem

Consider the three-slab concrete pavement system for which n surface deflections are measured at different locations. The three unknowns are the modulus of concrete slab E_c, the modulus of subgrade reaction k and the load-transfer efficiency JTE. Slab thickness and Poisson ratio are assumed to be known and the shear modulus of subgrade is assumed to be zero. For backcalculation of the three unknowns from the measured surface deflections, we have a system of simultaneous equations as follows :

$$f_1(x) = w_{m1} - w_{c1} = 0 \quad \text{..............................(3a)}$$

... ...

$f_i(x) = w_{mi} - w_{ci} = 0$ ....(3i)

... ...

$f_n(x) = w_{mn} - w_{cn} = 0$ ....(3n)

where w_{mi} and w_{ci} are the measured and calculated deflections respectively at sensor i, and x is the vector of the three unknowns, the slab modulus E_c, the modulus of subgrade reaction k and the load-transfer efficiency JTE.

Although the three-slab system gives analytical solutions for surface deflections, it is not possible to derive a closed-form procedure for backcalculating load-transfer efficiency an layer moduli. Iterative methods was employed to evaluate load-transfer efficiency on the basis of deflection measurements.

Since Eqs. 3 have more number of equations than the number of unknowns, the method of least-squares is used in the backcalculation analysis. Assume a system of m equations in n unknowns written as (Hahaner et al. 1989, Kincaid and Cheney 1991)

$$Ax = b \qquad \text{..(4)}$$

where A is an mxn matrix, x is an nx1 column vector and b is an mx1 column vector. The rank of A is assumed as n, m≥n.

Multiplying both sides of Eq. 4 by A^T, we have

$$A^TA\,x = A^Tb \qquad \text{......................................(5)}$$

where A^T is the transpose of matrix A. The equation in Dq. 5 are called normal equations.

The so-called least-squares "solution" is the vector x that makes the Euclidean norm $\|b-Ax\|_2$ a minimum. The direct use of the normal equations for solving the least-squares problem is very appealing due to its conceptual simplicity. However, it is generally accepted that this method is one of the least satisfactory methods for the least-squares problem. The reason is that the condition number of A^TA may be considerably worse than that of A and Eq. 4 may be ill-conditioned.

2 Modified Newton's Method

A method called modified Newton's method has been used in backcalculation successfully (Harichandran et al. 1992, Almeida et al. 1994). It is natural to use the Newton's method since the expression for deflections is implicit and it is impossible to compute the differential of deflection with respect to JTE or layer modulus analytically. Expanding Eq.3 into Taylor's series and taking the first-order terms, we can reduce Eqs. 3 into a system of linear equations $G\,\Delta E = \Delta w$, or rewritten as follows :

$$\begin{bmatrix} \frac{\partial f_1}{\partial x_1} & \frac{\partial f_1}{\partial x_2} & \frac{\partial f_1}{\partial x_3} \\ \cdots & \cdots & \cdots \\ \frac{\partial f_n}{\partial x_1} & \frac{\partial f_n}{\partial x_2} & \frac{\partial f_n}{\partial x_3} \end{bmatrix} \begin{bmatrix} \Delta x_1 \\ \Delta x_2 \\ \Delta x_3 \end{bmatrix} = \begin{bmatrix} \Delta w_1 \\ \Delta w_2 \\ \cdots \\ \Delta w_n \end{bmatrix} \qquad \text{...........(6)}$$

where Δwi is the value of $(w_{mi} - w_{ci})$, and Δx_1, Δx_2 and Δx_3 are the increments of E_c, k and JTE respectively, and $\partial f_i/\partial x_j$ is the first-order differential of $(w_{mi} - w_{ci})$ (i = 1, 2, ...n) with respect to x_j (j=1, 2 and 3) expressed as

$$\frac{\partial f_i}{\partial x_j} = \frac{w_{ci}(x_1,...x_j + \Delta x_j,... x_n) - w_{ci}(x_1, ...x_j, ...x_n)}{\Delta x_j} \qquad \text{................(7)}$$

The condition number of G^TG is equal to the square of the condition number of G. The direct use of Eq.5 poses problems such as divergence and the presence of local minima. The orthogonal factorisations or singular value decomposition methods are recommended to minimise the occurrence of the above two problems (Harichandran et al. 1992). The Householder Transformation is employed to solve Eq. 6 because it is effective in dealing with an ill-conditioned system of equations (Kahaner 1989, Kincaid and Cheney 1991, James et. al 1993).

The key part of Householder Transformation is to obtain an mxm matrix, P, to reduce G (mxn) into an upper triangular form. The procedure for deriving P is presented as follows :

$$P = I - 2\frac{vv^T}{v^Tv} \qquad \text{.........................(8)}$$

where v is a nonzero mx1 column vector. If "g" is any column of G, v cam be computed from

$$v = g \pm \|g\|_2 e_1 \qquad \text{.................................(9)}$$

where $\|g\|_2$ is the Euclidean norm of g, and $e_1 = (1, 0, 0, ...0)^T$. We can use $v = g + \|g\|_2 e_1$ or $v = g - \|g\|_2 e_1$. However, since subtraction may sometimes lead to round-off errors, it is safer to choose the sign "+".

3 The NUS-BACK4 Computer Programme

A computer programme names NUS-BACK4 was coded in Microsoft FORTRAN-77 to perform backcalculation based on the procedure described in the proceeding section. The backcalculation proceeds in the following sequence :

1) Read input data such as measured deflections (w_{m1}, w_{m2}, ... w_{mi}, ...w_{mn}) and initial values of layer moduli, E_c and k, load-transfer efficiency of joint, JTE.

2) Calculate the gradient matrix G(mx3) with respect to the initial values of E_c, k and JTE.

$$G = \begin{bmatrix} g_{11} & g_{12} & g_{13} \\ g_{21} & g_{22} & g_{23} \\ ... & ... & .... \\ g_{m1} & g_{m2} & g_{m3} \end{bmatrix} \quad ....(10)$$

where the element "g_{ij}" can be determined from Eq. 7. For example, g_{12} can be expressed as

$$g_{12} = \frac{w_{c1}(x^0{}_1,\ x^0{}_2\ (1+\gamma),\ x^0{}_3) - w_{c1}(x^0{}_1,\ x^0{}_2,\ x^0{}_3)}{\gamma x^0{}_2} \quad ....(11)$$

in which w_{c1} is calculated deflection at the location of sensor 1, $x^0{}_1$, $x^0{}_2$ and $x^0{}_3$ are the initial values of E_c, k and JTE, and γ is the incremental ratio which should be sufficiently small (normally from 0.001 to 0.01).

(3) Calculate the column vector $\Delta w = [(w_{c1}-w_{m1}),\ (w_{c2}-w_{m2}),\ ...(w_{cm}-w_{mm})]^T$.

(4) Solve the column vector Δx from $G\Delta x=\Delta w$.

(5) Let $x^1=x^0 + \Delta x$, and repeat step 2 to step 4 with respect to x^1.

(6) Terminate iteration if the changes in the layer moduli are sufficiently small, i.e.

$$\frac{x_i^{\ j+1} - x_j^{\ i}}{x_j^i} \leq \varepsilon \ (x^i = E_c,\ k, \text{ and JTE}) \quad ......(12)$$

where x_i^j and x_i^{j+1} are the values of layer moduli in the jth and (j+1)th iteration respectively, and ε is the tolerance for checking the changes in layer moduli.

Validation of NUS-BACK4 Using Theoretical Deflection Basins

1 Validation Analysis

Two pavement systems were employed to validate the NUS-BACK4 programme. The two pavements and their corresponding theoretical deflection basins are presented in Table 2. Two different basins were computed for each pavement system under central and edge loading respectively.

Edge Loading

Table 3 gives the backcalculating results with respect to the initial values of E_c, k and JTE under edge loading. It is shown that in Table 3, all backcalculation results converge to the true values for all initial values. In addition, the number of iterations depends on the initial values. When the initial values are closer to the true values, the backcalculation results generally converges to the true values after 5 iterations.

Central Loading

Table 4 gives the backcalculation results with respect to different initial values E_c, k and JTE under central loading. The following observations can be made :

1) The final solution is very sensitive to the initial vale of JTE. This may be attributed to the fact that small changes in deflections can result in great changes in JTE in the inverse solution.

2) Backcalculation results may converge to false values as found in case I. The negative values of JTE indicate that the unloaded slabs deflect upward, and are in contradiction with its actual values varying from 0 to 2.0.

3) Iterative calculation may diverge as indicated in case II.

4) If the initial values are properly estimated, backcalculation results will converge to the true values after 5 iterations.

2 Issues in Iterative Backcalculation Methods

It is known that the solution of an interative backcalculation algorithm is affected by the initial values of parameters selected. All existing backcalculation programmes based on the iterative methods suffer from two potential problems. One is the nonuniqueness of the final solution and the other is the divergence of answers. These are also observed in the NUS-BACK4 solutions.

The nonuniqueness and the divergence problems arise because the multi-dimensional surface represented by the least-squares problem may have many local minima, and thus more than one set of pavement parameters can generate a similar deflection basin. There are two ways which may lessen the two problems. One is by providing an accurate estimation of the initial values. Some programmes have been reported capable of estimating the initial values (Harichandran et at. 1992). Normally, the estimation of initial values are made through experience, such as empirical equations developed through regression analysis of a wide range of pavements with various pavement conditions. A second way is by using the orthogonal factorisations to solve the least-

Table 2 Theoretical Basins for Two Pavements

Case	Slab Size	Loading Position	Deflection (10^{-3} mm) r=0mm	r=300mm	r=600mm	r=900mm
I	600x600 mm	Central	150.582	138.222	118.463	97.371
		Edge	283.337	229.828	175.864	130.223
II	600x600 mm	Central	115.783	104.118	87.207	69.953
		Edge	182.147	148.174	111.371	80.894

Table 3 Backcalculation Results under Edge Loading

Case	Initial Value E_c(GPa)	k(MN/m³)	JTE	Deflection (10^{-3} mm) E_c(GPa)	k(MN/m³)	JTE	Iterations
I	5	10	0.01	30.0	50.0	0.60	5
	15	20	0.01	30.0	50.0	0.60	5
	20	30	0.01	30.0	50.0	0.60	5
	40	80	0.90	30.0	50.0	0.60	5
II	5	10	0.01	40.0	150.0	0.90	8
	15	20	0.01	40.0	150.0	0.90	7
	20	30	0.01	40.0	150.0	0.90	5
	50	200	1.0	40.0	150.0	0.90	5

Table 4 Backcalculation Results under Central Loading

Case	Initial Value E_c(GPa)	k(MN/m³)	JTE	Backcalculation Result E_c(GPa)	k(MN/m³)	JTE	Iterations
I	5	10	0.01		Diverged		
	5	10	0.10	30.0	51.7	-0.20	7
	15	20	0.01	30.0	51.7	-0.20	8
	15	20	0.10	30.0	50.0	0.60	5
	20	30	0.01	30.0	51.7	-0.20	17
	20	30	0.10	30.0	50.0	0.60	5
II	5	10	0.01		Diverged		
	5	10	0.10	40.0	150.0	0.90	5
	20	30	0.10		Diverged		
	20	30	0.50		Diverged		
	20	100	0.10	40	150.0	0.90	5
	30	30	0.10		Diverged		
	50	200	0.10	40.0	150.0	0.90	5

squares method (Harichandran et at. 1992). The Householder transformation is one of such methods.

For the specific backcalculation problem of a three-slab pavement system, it appears that the use of deflection basins under edge loading may also lessen the nonuniqueness and divergence problems. It is shown that in Tables 3 and 4, the backcalculation results are more sensitive to initial values under central loading than under edge loading. The reason is that, as discussed in section 2.2, the effect of JTE on deflections is very small in the case of central loading.

Conclusions

This paper describes the development of backcalculation algorithm NUS-BACK4. A computer programme named Slab3-1 developed on the basis of the three-slab model by Shi (1995) was incorporated in the NUS-BACK4 as a subroutine for solving analytical deflections. The backcalculation scheme was derived based on the least-squares method. The Householder transformation was employed to search for the final solutions.

The effect of load-transfer efficiency of joints on deflections depends on the radius of relative stiffness, slab size and loading position. If the slab size is small (300x300 cm) and the radius of relative stiffness is low (0.5 m), the effect of load-transfer efficiency varies from 0 to 1.0 under central loading. The deflections under edge loading vary significantly with respect to load-transfer efficiency of joints. A variation of about -50% in deflections is found when the joint transfer efficiency varies from 0 to 1.0 regardless of slab sizes.

Like other iterative backcalculation algorithms, the non-uniqueness and divergence problems hamper the productiveness of the NUS-BACK4 programme. However, the two problems can be reduced by providing a proper estimation of the initial values. The Householder transformation can be used to lessen the two problems. In addition, the use of deflection measurements under edge loading may also reduce the two problems since the deflections are very sensitive to JTE and the effect of round-off errors can be reduced.

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